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COMPARATIVE DESIGN OF REINFORCED CONCRETE SHEAR WALLS REGARDING DUCTILITY AND BUILDING CODES REQUIREMENTS

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ABSTRACT

Designers are often faced with the question of which ductility category the reinforced concrete shear walls must be designed for. Code provisions for the design of reinforced concrete shear walls are intended to provide adequate reinforcement details and concrete strength to permit inelastic response under major earthquakes without critical damage or collapse. However, where the choice of building code depends on the jurisdiction, the choice of ductile v/s moderately ductile shear wall depends on the designer. It is desirable from the economic, safety, and design effectiveness point of view to use the most optimum ductility factor R_d. This study uses primarily the Canadian CSA A23.3 design standard along with an overview over the American ACI 318-08, and the New Zealand NZS 3101-2006 design provisions to compare the requirements of the shear wall design using different ductility approaches. A comparative analysis of various ductility requirements that includes a numerical application is provided. The design includes computer modelling using ETABS and the detailing of the shear walls obtained from each. The design procedure obtained, resulted in a more economical design in the case of conventional construction for low-rise buildings, and moderately ductile for medium-rise building.

Introduction

Shear walls are structural elements that resist, mainly by vertical cantilevered flexural action, the lateral forces acting parallel to the plane of the wall. Under seismic cyclic reversed loads, and due to the confinement effect of the reinforcement cage on the cracked concrete, shear walls are capable of withstanding flexural plastic hinging without failing in shear. That however, depends on the ductility level of the shear wall. Code provisions for the determination of earthquake loads are intended to give a reasonable estimate of the lateral forces that act on a building as a result of an earthquake. Design standards such as CSA A23.3-04, ACI 318-08, and NZS 3101-2006, provide special seismic provisions for the design of shear walls. These can be characterized in three different design philosophies: (i) conventional construction design, (i) nominal ductility design, and (iii) ductile wall design. Designers are often faced with the

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decision of which design philosophy should be undertaken to achieve the optimized shear wall design in terms of economy and structural performance.

The objective of this study is to provide a comparative overview of various building code requirements for the determination of the earthquake design loads, and for the design of reinforced concrete shear wall with various ductility factors. A brief overview of the requirements of three design standards listed above will be provided. In addition, two structures of low and medium-rise will be analysed in this study in each of which shear walls will be designed for three ductility factors in accordance with CSA 23.3-04 design standard. The analysis will contribute to a better understanding of the code requirements regarding ductility of shear walls and their implications on the design. The analysis will also help to determine which design approach is the most economical and effective based on both the geographic location and the characteristics of the building. This study is an attempt to provide for the structural designers a procedure to utilize the building codes and design standards requirements regarding ductility in the most optimized way from the point of view of economy, and effectiveness.

Description of the Analysis Models

To evaluate the impact of building heights and over-all stiffness on shear walls ductility design, two reinforced concrete shear wall structures were analyzed, a 15-storey structure, and a 6-storey structure. The selection of building heights in the two structures was made in such way that the 15-storey structure remains within the 60m-height limit, and the 6-storey structure remains within the 20m-height limit, so that the Equivalent Static Force Procedure Analysis procedure may be applied by NBCC 2005 Cl.4.1.8.7.1 to both structures.

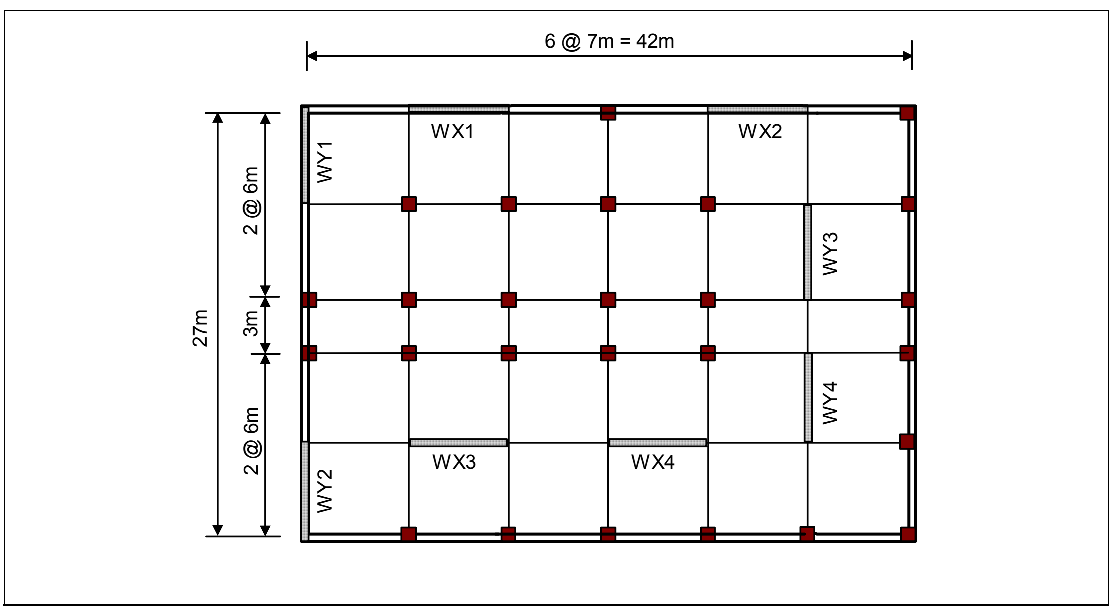

Figure 1. Typical floor layout of building.

Fig. 1 represents a typical $1135m^2$ floor plan used for both structures. The shear wall locations in this layout were selected randomly in the first run of the analysis. In this assumed office building located in Ottawa, Ontario, and founded on dense soil or soft rock, 200 mm thick flat slabs were used in the analysis with a super-imposed dead load of 1.2 kPa. Assuming a large area is allocated for corridors, the live load is taken as 4.8 kPa. 25% of Snow load on the roof is included in the seismic design. Since shear walls must also resist factored axial loads due to gravity, the load combination of load case 5 in Table 4.1.3.2 of NBCC 2005 was used:

$$1.0D + 1.0E + 0.5L + 0.25S \quad (1)$$

Using the climatic data of the supplement SB-1 of NBCC 2005 for Ottawa, Ontario, we have: $S_a(0.2) = 0.66$, $S_a(0.5) = 0.32$, $S_a(1.0) = 0.13$, $S_a(2.0) = 0.044$, and Peak Ground Acceleration = 0.42. Since it was assumed that the structure is founded on dense soil or soft rock, a site class C is appropriate for this design, and the acceleration-based and velocity-based site coefficients F_a, and F_v are both determined as 1 according to Table 4.1.8.4.B of NBCC 2005. The design spectral acceleration for the structure is determined in accordance with NBCC 4.1.8.4.6.

NBCC 2005 defines the fundamental lateral period of the structure T_a as $0.05(h_n)^{0.75}$ for shear wall structures (NBCC 2005, Cl.4.1.8.11.3). However, NBCC allows T_a to be determined using structural modeling that represents the mass and stiffness distribution of the structure including stiff elements that are not part of the Seismic Force Resisting System. In all cases, T_a must not be greater than 2 times $0.05(h_n)^{0.75}$ for shear wall structures (NBCC 2005, Cl.4.1.8.11.3). In order to do the analysis using the Equivalent Force Static Procedure, the lateral period of the structure must not exceed 2.0 sec (NBCC 2005, Cl. 4.1.8.7). The structural models considered in this study were analyzed using ETABS version 9.6.0 , where two separate finite element runs were considered for each case of analysis. First the model was run including both the SFRS, which in this case are reinforced concrete shear walls, and the columns as fixed at both ends. The second run of the model didn't include the columns. The purpose was to estimate a realistic lateral period of the structure from the first run, and input that value for the lateral period in the second run without the columns and design the shear walls for 100% of the resulting seismic forces. In addition, the effects of cracked sections in reinforced concrete were taken into account using bending stiffness modifiers of 0.25 for slabs, and 0.7 for columns and walls. These assumptions are in agreement with CSA A23.3-94 Cl.10.14.1.2 and ACI 318-08 Cl.10.10.4.1.

The static base shear is obtained from the procedure NBCC 2005 Cl. 4.1.8.11 as:

$$V = S(T_a)M_v I_E W / (R_d R_o) \quad (2)$$

And,

$$S(2.0)M_v I_E W / (R_d R_o) < V < (2/3)S(0.2)\, I_E W / (R_d R_o) \quad (3)$$

15-Storey Structure

Using the floor layout of Fig. 1, for this 15-storey structure, the typical storey height is 3.8 m, and the total building height is 57 m. Shear walls are 500mm thick, 7m long in the E-W direction, and 6m long in the N-S direction. Columns are assumed to be 600×600. The weight of the structure was estimated to be approximately 150,000 kN including the self weight of the slabs, walls, columns, the super-imposed dead loads, and 25% of the roof snow load. For shear wall structures, the fundamental period according to NBCC 2005, Cl.4.1.8.11.3(c) is $0.05\times(57)^{0.75}$ = 1.04 sec. The analysis of the first run of the model, the lateral periods of the structure modeled with SFRS and columns that are fixed, obtained from ETABS were as follows: (i) in the North-South direction, T_{N-S} = 3.2 sec, and (ii) in the East-West direction, T_{E-W} = 2.81 sec. However, as stated above, since the lateral period of this structure must not exceed 2×1.04 = 2.08 sec and must also not exceed 2.0 sec in order to use the Equivalent Static Force Procedure, a period of 2.0 sec achieves highest possible flexible structure in this case.

For $S_a(0.2) / S_a(2.0) = 0.66 / 0.043 = 15.35$, the higher mode factor M_v, and the Base Overturning Reduction Factor J are determined to be 2.5 and 0.4 respectively, as specified in NBCC 2005 Table 4.1.8.11. Since at T_a = 2.0 sec, $S(T_a) = S(2.0)$, the static base shear will be at its minimum allowed by NBCC as shown by Eq. 3. Therefore, the design earthquake forces will be at their minimum for any period of the structure of 2.0 sec and higher. Substituting in Eq. 2 and in Eq. 3 for S(2.0) = 0.044 we have:

$$V_{15\text{-Storey}} = V_{min} = 0.11\ W/\ (R_dR_o) \tag{4}$$

In this analysis, the optimum flexibility of the structure is achieved by using the highest lateral period of the structure that NBCC 2005, which resulted in the minimum static base shear, applied on the building.

6-Storey Structure

Using the same floor layout and storey height as previously for a 6-storey structure, the total building height is 19 m. The shear walls are considered 350mm thick, and the columns 350×350. The weight of the structure was estimated to be approximately 43,000 kN including the self weight of the slabs, walls, columns, the super-imposed dead loads, and 25% of the roof snow load. The fundamental period according to NBCC 2005, Cl.4.1.8.11.3(c) is $0.05\times(19)^{0.75}$ = 0.45 sec. The lateral periods of the structure obtained from modal analysis with ETABS were as follows: (i) in the North-South direction, T_{N-S} = 0.56 sec, and (ii) in the East-West direction, T_{E-W} = 0.47 sec. Those values are very close to the empirical code equation, which indicates no opportunity to increase the lateral period of the structure.

For $S_a(0.2) / S_a(2.0) = 0.66 / 0.043 = 15.35$, the higher mode factor M_v, and the Base Overturning Reduction Factor J are determined to be both equal 1.0 as obtained from NBCC 2005 Table 4.1.8.11. At the lateral period obtained, response spectrum acceleration obtained S(0.45) = 0.27. Therefore we have:

$$V_{6\text{-Storey}} = 0.19\ W/\ (R_dR_o) \tag{5}$$

Second Analysis With 50% Fewer Walls

A second analysis was performed with a more flexible structure by randomly removing approximately 50% of the shear walls up to the limit of not exceeding the maximum allowable interstorey drift of 2.5% of the storey height, calculated with $R_dR_o = 1.0$ (NBCC 2005 Cl. 4.1.8.13). Table 1 below summarizes the lateral periods obtained for each structure, for both the original layout and the more flexible analysis with fewer walls. Table 1 also shows the interstorey drifts obtained that were kept below the 2.5% limit.

Table 1. Lateral periods and interstorey drifts obtained for both structural models.

Direction	Lateral Period Obtained by Modal Analysis	Largest Interstorey Drift	Lateral Period Obtained by Modal Analysis	Largest Interstorey Drift
	Original Layout (15-Storey)		*50% Less Walls (15-Storey)*	
N-S	3.2 sec	1.69%	3.67 sec	2.5%
E-W	2.81 sec	1.35%	3.4 sec	1.96%
	Original Layout (6-Storey)		*50% Less Walls (6-Storey)*	
N-S	0.56 sec	0.46%	0.77 sec	0.89%
E-W	0.47 sec	0.29%	0.63 sec	0.54%

Several trial layouts were analyses in order to reach the highest interstory drift not exceeding 2.5% to achieve the most flexible layout practically possible. For the 15-storey structure, the lateral periods of the second analysis obtained from the ETABS model were: (i) in the North-South direction, $T_{N\text{-}S} = 3.67$ sec, and (ii) in the East-West direction, $T_{E\text{-}W} = 3.4$ sec. This much more flexible structure, with the exception of the minor effect of building weight reduction due to less wall mass, did not impact the static base shear required by code since the period of the structure cannot be higher than 2.0 sec as described above. The seismic forces will, however, be distributed among less walls. For the second analysis of 6-storey structure, the lateral periods of the second analysis obtained from the ETABS model were less than twice the value of the code empirical equation of 0.45 sec but, nevertheless, allowed the static base shear to be reduced. In the East-West direction for example, $T_{E\text{-}W}$ obtained was 0.63 sec. This resulted in a static base shear that equals 0.14 W/ (R_dR_o) which is less than the static base shear of Eq. 5 obtained above. The seismic forces will, however, be distributed among less walls and the analysis that follows should show that they would in fact increase on all the walls.

Design Standards Requirements

Building codes in high seismic areas have similar requirements to take into account the ductile behaviour of shear walls. For example, according to CSA A23.3-04, the minimum reinforcement required for conventional structural walls is $0.0015A_g$ for vertical reinforcement and $0.002A_g$ for horizontal reinforcement with spacing not exceeding 500mm or 3 times the wall thickness, where A_g is the gross concrete section area. Concentrated zone reinforcement requirement for conventional walls is a minimum of 2-15M verticals at both ends. (Cl.14.1.8). With ACI 318-08, the minimum reinforcement requirement is $0.0025A_g$ for all types of ductility in all zones of the wall (Cl.21.9.2). To account for ductility, ACI requires special boundary elements whenever the depth of the neutral axis exceeds a certain critical value that is related to the wall displacement under the factored loads (Cl.21.9.6). In NZS 3101-2006 the minimum reinforcement requirement is $\sqrt{f_c'} / 4f_y$, for example, with f_c' = 25 MPa, and f_y = 400 MPa, the minimum A_s would be $0.0037A_g$ which is higher than the minimum steel requirement of CSA and ACI. However, NZS allows the use of lower than that ratio if the reinforcement is more than one third greater than that required by the analysis as long as it's greater than $0.0014A_g$ or $0.7A_g / f_y$ (Cl.11.3.11.3). At higher ductility, unlike CSA, NZS does not have different minimum reinforcement requirements for higher ductility but rather additional steel detailing, anchorage and ties requirements. CSA requires at higher ductility (R_d = 3.5) a concentrated reinforcement zone tied as columns in which horizontal reinforcement are anchored, and in which the minimum reinforcement required is $0.0015A_g$ in plastic hinging region, or $0.001A_g$ outside the plastic hinge region, placed in two layers with a minimum of 4 bars (Cl.21.6.6.9). For the distributed reinforcement, CSA A23.3-04 requires a minimum reinforcement of $0.0025A_g$, which is the same as that required by ACI. (Cl.21.6.5.1). For shear capacity, the maximum shear demand must not exceed a value between $0.10\phi_c f_c' b_w d_v$ and $0.15\phi_c f_c' b_w d_v$ depending on the inelastic rotational demand of the wall, which is rotation of the wall relative to the centre of the plastic hinge due to inelastic deformations (Cl.21.6.9.6). ACI requires a maximum factored shear demand on all types of shear walls designed to resist seismic loads (Cl.21.9.4). Similarly, NZS 3101-2006 also requires a maximum factored shear demand based on the shear strength provided by the concrete (Cl.11.4.7.3).

In the following sections, the wall with the most sever seismic loads will be considered as an example study which in this case is found to be wall WX3. The reinforcement detailing of the wall is determined following the procedure shear wall design for the different ductility requirement of CSA A23.3 Cl.21.8. Applying the load combination of NBCC 2005 presented in Eq.1, for gravity, and earthquake forces on the wall, The factored axial load is determined to be 16,220 kN for the 15-storey building and 4,772 kN for the 6-storey building. The seismic forces are determined by running the analysis model with ETABS for up to 12 mode shapes that achieved over 90% modal participation mass ratio. Rigid diaphragms were assumed. The analysis is carried out using: (i) the conventional construction approach (R_d = 1.5, R_o = 1.3), where a lateral base shear of 8460 kN, and a total overturning moment of 330,000 kN.m where obtained, (ii) the moderate ductility approach (R_d = 2.0, R_o = 1.4), where a lateral base shear of 5892 kN, and a total overturning moment of 230,000 kN.m were obtained, and (iii) the ductile walls approach (R_d = 3.5, R_o = 1.6), where a total lateral base shear of 2946 kN, and a total overturning moment of 115,000 kN.m were obtained.

Results

Table 2 below, summarizes all the design forces and resistance moments and shear, in addition to the utilization factors and steel quantities for Wall WX3 for the 15-storey structure. Wall WX3 was designed to resist the gravity and seismic forces obtained for each ductility factor as required by Chapter 21 of CSA23.3-94. Shear resistance of the wall for R_d = 1.5, 2.0, and 3.5, is required to be magnified by the lesser of the ratio of the flexural capacity of the wall M_r / M_f or R_dR_o (Cl.21.8.3.2), the lesser of the ratio of the nominal flexural capacity of the wall M_n / M_f or R_dR_o (Cl.21.7.3.4.1), and the lesser of the ratio of the probable flexural capacity of the wall M_p / M_f or R_dR_o (Cl.21.6.9.1), respectively.

Table 2. Design results for the 15-storey structure.

15- Storey	$R_d = 1.5$		$R_d = 2.0$		$R_d = 3.5$	
15- Storey	1st Model (Original Layout)	2nd Model (half the walls)	1st Model (Original Layout)	2nd Model (half the walls)	1st Model (Original Layout)	2nd Model (half the walls)
V_f (kN)	2532	4544	1763	3165	882	1582
M_f (kN.m)	35560	66372	24765	46224	12382	23112
M_r (kN.m)	48058	70731	54656	54656	59083	59083
M_n (kN.m)	55462	82090	63597	63597	69083	69083
M_p (kN.m)	57285	90466	67389	67389	74211	74211
Magnified V (kN)	3422	4842	4527	4355	4939	5080
V_r (kN)	4541	4989	4781	4781	5350	5350
Flexural utilization	0.74	0.94	0.45	0.85	0.29	0.39
Shear utilization	0.75	0.97	0.95	0.91	0.92	0.95
Steel Quantity (kg/m)	122	309	201	201	282	282

Fig. 2 shows one example of detailing for the analysis of the 15-storey structure with approximately 50% fewer shear walls obtained for Wall WX3, the reinforcement detailing is shown for conventional construction design, moderate ductility design, and for ductility design respectively.

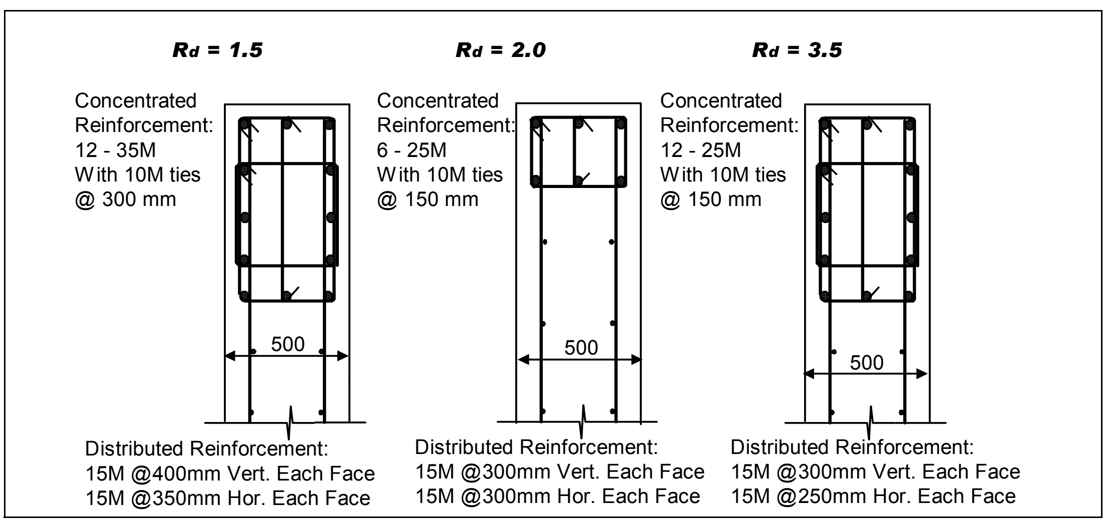

Figure 2. Wall section reinforcement detailing for 15-storey building with 50% fewer walls.

Table 3 below, summarizes all the design forces and resistance moments and shear, in addition to the utilization factors and steel quantities for Wall WX3 for the 6-storey structure.

Table 3. Design results for the 6-storey structure.

	R_d = 1.5		R_d = 2.0		R_d = 3.5	
6- Storey	1st Model (Original Layout)	2nd Model (half the walls)	1st Model (Original Layout)	2nd Model (half the walls)	1st Model (Original Layout)	2nd Model (half the walls)
V_f (kN)	2537	3396	1767	2365	883	1182
M_f (kN.m)	34186	46967	23808	32709	11904	16355
M_r (kN.m)	34432	47814	26833	32902	32398	32398
M_n (kN.m)	39497	55745	29929	37332	36838	36838
M_p (kN.m)	45126	65113	33466	42543	41920	41920
Magnified V (kN)	2555	3457	2220	2720	3109	3031
V_r (kN)	3179	3668	3049	3049	3895	3895
Flexural utilization	0.99	0.98	0.89	0.99	0.37	0.5
Shear utilization	0.80	0.94	0.73	0.89	0.8	0.78
Steel Quantity (kg/m)	219	358	148	220	235	235

All the reinforcement detailing is summarized in Table 4 below, where the reinforcement detailing obtained for Wall WX3 for the two structures, for the three ductility approaches, and for the two types of analysis each. The first model is the analysis with the original layout, and the second model is the analysis with half the walls deleted, where walls WX1, WX4, WY2, and WY3 all removed from the building layout of Fig. 1. In the case of the 15-storey building, the wall in question WX3 was designed for higher seismic forces in the second analysis than in the first analysis. While this was reflected in the detailing for conventional construction, there was no difference in the wall detailing for moderately ductile, and ductile design due to the benefit of the lower required forces obtained on the higher ductility walls.

Discussion and Conclusions

The results show that conventional construction design for R_d = 1.5 was the most economical option in the first stiffer building layout, followed by the moderate ductile design. Ductile walls did not appear to be a feasible design option for that building. Due to the low flexural utilization of the ductile walls, shear magnification resulted in a high amount of total

Table 4. Shear wall reinforcement summary.

	$R_d = 1.5$		$R_d = 2.0$		$R_d = 3.5$	
15- Storey 500mm Walls	1st Model (Original Layout)	2nd Model (half the walls)	1st Model (Original Layout)	2nd Model (half the walls)	1st Model (Original Layout)	2nd Model (half the walls)
Concentrated Reinforcement	2 – 15V	12 – 35V w/ 10ties @300	6 – 25V w/ 10ties @150	6 – 25V w/ 10ties @150	12 – 25V w/ 10ties @150	12 – 25V w/ 10ties @150
Distributed Reinforcement	15@400 HEF 15@350 HEF	15@400 HEF 15@350 HEF	15@300 HEF 15@300 HEF	15@300 HEF 15@300 HEF	15@300 HEF 15@250 HEF	15@300 HEF 15@250 HEF
Quantity (kg/m)	122	309	201	201	282	282
6- Storey 350mm Walls	1st Model (Original Layout)	2nd Model (50% Less Walls)	1st Model (Original Layout)	2nd Model (50% Less Walls)	1st Model (Original Layout)	2nd Model (50% Less Walls)
Concentrated Reinforcement	12 – 25V w/ 10ties @300	20 – 30V w/ 10ties @300	6 – 25V w/ 10ties @150	12 – 25V w/ 10ties @150	10 – 25V w/ 10ties @150	10 – 25V w/ 10ties @150
Distributed Reinforcement	20@350 HEF 15@500 HEF	20@350 HEF 15@400 HEF	15@450 HEF 15@450 HEF	15@350 HEF 15@450 HEF	15@300 HEF 15@300 HEF	15@300 HEF 15@300 HEF
Quantity (kg/m)	219	358	148	220	235	235

reinforcement (282 kg/m compared to 122 kg/m for conventional construction and 201 kg/m for moderately ductile wall). However, it seems that moderately ductile walls were the optimal option for a building of this height when a more flexible and less stiff layout was used, even though that static base shear remained the same for all three designs in each analysis.

The findings of this study contribute to develop a procedural approach in determining the optimum ductility design of reinforced concrete shear walls. The results indicate that ductile walls are not necessarily the most economical design even in the case where the structure reaches the maximum flexible period allowed by the design code. At some level of flexibility, the structural design was more economical with the conventional construction approach in the first analysis. This could be the better approach where structural designers have limited options as to the number and locations of walls in the structure due to the presence of elevator and stair cores and other architectural constraints where walls cannot be eliminated. However, in the case where reducing the number of walls to the most optimum design is possible, the design could be more economical using moderate ductility or higher ductility approach. In this example, the conventional construction approach was the most economical in the first analysis where a larger number of walls existed in the layout, while the moderate ductility approach was the most economical in the second analysis where walls were removed. The higher ductility approach could become the most economical in taller building where the flexural utilization factor is even higher; and as a result, the shear magnification required will be lower.

It should be noted that Table 4.1.8.9 of NBCC 2005 does not permit the conventional wall design system for buildings height over 30m where $I_E F_v S_a(1.0)$ is higher than 0.3 which is the case for the 15-storey building of this study. However, the conventional wall design was carried out for the purpose of the analysis. It should also be noted that some of the main elements of the analysis weren't carried out in this study, such as the capacity of the columns to resist the additional drift due to the removal of walls, and the fact that the wall reinforcement should be maintained through the height of the plastic hinges affects the comparison of steel quantities. Another limitation of this study is the fact the SFRS system was not verified to resist wind loads.

This study illustrated some of the implications of code requirements on the ductility design of shear walls. The benefits of higher lateral periods on the design reaches a limit due to code requirements such as the minimum concentrated reinforcement requirements of CSA A23.3 in ductile shear walls which makes the design un-economical at low flexural utilization for capacity design. The ductile walls approach could be more economical if the CSA standard allowed some reduction in the minimum reinforcement requirement under certain conditions, such as where the reinforcement provided is a lot greater than what's required by the analysis. The NZS standard allows such reduction in the minimum reinforcement requirement as described before in this study. The findings of this study contribute to a better understanding of the economical implications of the various building code requirements regarding ductility design. While the behaviour of shear walls is largely inelastic, and the intent of the building codes might be to ensure ductile behaviour when it's desired, the various building code requirements have the implications of allowing the decision of the ductile approach to be taken based on economy.

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